Seismic Analysis of Beam-Type Bridges in Sudan

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مُسْتَخْلَص

يتناول موضوع البحث دراسة التحليل الزلزالي لجسور العارضات بالسودان.

تناول البحث بصورة تفصيلية دراسة حالة جسري بري والمنشية الذَين تم تشييدهما في ولاية الخرطوم.

الجسور المختارة تمثل جسور الخرسانة مسبقة الإجهاد الصندوقية و الجسور التي على شكل I .

تم إجراء التحليل الزلزالي بطريقتي مخططات طيف الاستجابة وطريقة التكامل المباشر لمعادلة الحركة.

في هذه الدراسة تم بناء برنامج للحاسوب بلغة ماتلاب للتحليل ومن ثم طبق البرنامج على الحالات مثار البحث. تم استخدام برنامج SAP2000 لمقارنة النتائج وقد وجد أنه يعطي نتائج مطابقة للنتائج المتحصله من البرنامج الذي صممه الباحث. وجد ان البرنامج الذي تم تطويره يتسم بالفاعلية والاعتمادية وسهولة التطبيق.

وجد في البحث عند التحليل الزلزالي للجسور أن استجابة الجسر من حيث الإزاحة وقوى القص وعزوم الانحناء تتناسب تناسباً طردياً مع أقصى تسارع أرضي (PGA).

ABSTRACT

In this research seismic analysis of beam-type bridges in Sudan was studied.
Burri and Al Manshia Bridges were taken as case studies to investigate seismic effects. The two bridges were constructed at Khartoum State. The selected bridges represent prestressed concrete box and I-Girder beam-type bridges.
The seismic analysis was performed by two methods, direct integration time history analysis and response spectrum analysis.
A computer program has been developed in MATLAB and was used to analyze the problem under research. The Commercial Structural Analysis SAP2000 program [11] was also used for the comparative study and it was found that it gave typical results. The developed software has been found to be efficient, reliable and easy to apply.
It was found that bridge response in terms of displacements, shear and moments was directly proportioned to the peak ground acceleration (PGA).

Keywords: seismic analysis, beam-type bridges, response spectrum analysis

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1 Introduction

Seismic loads constitute a major component of environmental forces that all structures must be able to resist.

Bridges constitute one of the most important links in the lifeline of the modern world; the others are utilities carrying electricity, gas, and water, telecommunications, roads, and hospitals. It is therefore extremely important that bridges be built earthquake safe and be in a condition to serve as such essential links at all times.

Inspite of earthquake–induced damages to bridges worldwide, relatively little has been written or done in the way of research for the earthquake-resistant design of bridges.

Sudan is generally considered a country of low seismic activity. However; recent seismic activities in different regions within the Sudan warrant seismic hazard assessment of the Sudan. The country and its vicinity experienced one of the largest earthquake in recent history. The May 20,1999, 7.4 earthquake and its aftershocks that hit Southern Sudan is the one of the largest in continental Africa in the instrumental era of earthquake recording. In additional to the Southern Sudan, major portions in Central Sudan also experienced earthquake recently (e.g. Earthquakes stroke Kordofan State in August 1, 1993 with a magnitude of 5.5 and in November 15, 1993 with a magnitude of 4.3) [7]. Central Khartoum is affected by all seismic sources in Sudan and its vicinity though some sources, e.g. Kordofan State sources, are more sensible in Central Khartoum (Mohamedzein et al [7]). Alluvial deposits known locally as Gezira Formation underlie Central Khartoum. This formation includes a hard crust of fine grained soils underlain by saturated loose to medium dense sand. Given the recent earthquake activities and the vulnerable soil condition, an amplification of earthquake acceleration or soil liquefaction may occur in Central Khartoum. This fact is not appreciated by current design practice in Sudan. This is true regardless of the large amounts of investment in buildings and structures in Sudan as a whole and especially in Central Khartoum.

1.1 Aims and Objective

The objective of this research is to focus on seismic analysis of Sudanese bridges, especially beam type bridges.

The aim of this study could be summarized in the following:

- To review methods of analysis which can be used to solve beam type bridges and to select a method that is suitable for computer implementation.
- To put the selected method in a computerized form and build a computer program in a live language.
- To apply the developed computer program to solve an actual case study of beam type bridges.
- To study the results which have been obtained from the computer program, comment on them and give suggestions and recommendations.

2 Methodology

Finite Element models for the two bridges were built.

The seismic analysis was performed by two methods, direct integration time history analysis and response spectrum analysis.

The Response Spectrum for Central Khartoum, Sudan proposed by Y. Mohamedzein, et al. [7] was considered for the analysis.

A SIMQKE [6] program was used to artificially simulate earthquake motions compatible with the prescribed response spectra.

A computer program has been developed in MATLAB and was used to analyze the problem under research.

3 Burri Bridge

3.1 Bridge Description

Burri Bridge was constructed on the Blue Nile River at Khartoum in the year 1972. It is a perpetual bridge built with reinforced concrete with diversified applications. The total length of the bridge is 519.6m. The spans of the main bridge are 43.3+5×86.6+43.43, with prestresed non-uniform hinged T-shaped rigid frame box beams. The balanced cantilever part of the bridge is composed of six segments carried on six piers; Figure 1 shows the dimensions of a typical segment.

The width of the bridge deck is 22.82 m, and the bottom width of the single box is 5.14m. The height of the beam varies from 5m at the root of the cantilever to 1.6m at the end of the cantilever. The concrete box section has 120 bundles of prestressing steel tendons, each bundle is composed of 12D12.7, single strands 12φ7.

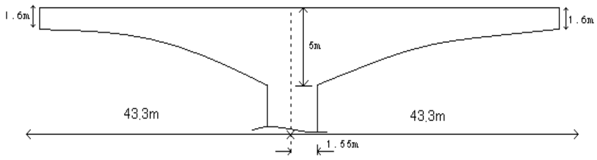

Figure 1: Typical Balanced Cantilever Segment in Burri Bridge

3.2 Cross Section Properties

The actual dimensions of the cantilever span of Burri Bridge are shown in Figure 2 and Figure 3. The total cross sectional area at various locations can be determined from the varying cross section by the following equation:

$$A = 13.6 + 1.6y \qquad (1)$$

where:

y = the depth of the variable part of the section, in meters, determined according to the following equation, assuming parabolic variation of cantilever depth,

$$y = 3.40 - 0.1629x + 0.00196x^2 \qquad (2)$$

where x denotes distance from support centerline. Values of x are calculated at 4.33m length intervals.

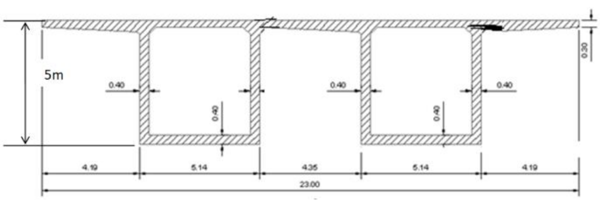

Figure 2: Actual cross section at the root of cantilever-Burri Bridge

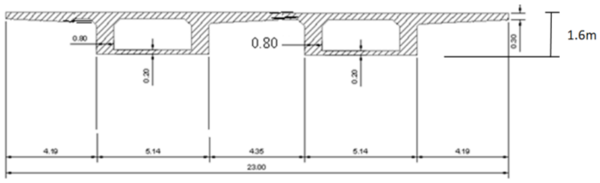

Figure 3: Actual cross section at the end of cantilever-Burri Bridge

3.3 Loading Test on Burri Bridge

Experimental dynamic and static load tests were conducted on Burri Bridge in the year 2005 by a Chinese Road and Bridge Company contracted by the Ministry of Physical Planning and Public Utilities, Khartoum State. The load test was performed as part of the assessment of the bridge prior to its maintenance. From the dynamic load test it was found that the natural vibration frequency $f_0 = 1.3623$ Hz and the Damping ratio D=.0268.

3.4 Case Study model

A finite element model is setup for the bridge span tested by the Chinese company. Actual cross section prosperities are used as noted in section 3.1-3.2.

Concrete compressive strength: According to the Chinese test report, the compressive strength of the main bridge was predicated, using Schmitt Hammer Test, to be 47.4 MPa after 30 year of service.

The modulus of elasticity of concrete was computed by the following equation, according to ACI Code:

$$E_c = 4700 \times \sqrt{f_c'} \qquad (3)$$

f_c' = cylinder specified strength of concrete (MPa) = 0.8 f_{cu}

f_{cu} = specified cube strength of concrete (MPa)

$$f_c' = 0.8 \times 47.4 = 37.92 \text{ MPa} \qquad (4)$$

A conservative value of 35 MPa will be used.

$$E_c = 4700 \times \sqrt{35} = 27805.5 \text{ MPa} \qquad (5)$$

3.5 Damping matrix:

According to the dynamic test a damping of ξ= 0.0268 will be considered [19].

$$C = a_0 M + a_1 K \qquad (6)$$

a_0, a_1 proportional factors determined from specified damping ratios ξ_i and ξ_j for the ith and jth modes, respectively.

$$\frac{1}{2}\begin{bmatrix} \frac{1}{\omega_i} & \omega_i \\ \frac{1}{\omega_j} & \omega_j \end{bmatrix}\begin{Bmatrix} a_0 \\ a_1 \end{Bmatrix} = \begin{Bmatrix} \xi_i \\ \xi_j \end{Bmatrix} \qquad (7)$$

For this case we take 1st mode and 5th mode

From modal analysis $\omega_1 = 9.9031\ rad/sec$ and $\omega_5 = 197.6140\ rad/sec$

Substitute in Eq (7)

$$\frac{1}{2}\begin{bmatrix} \frac{1}{9.9031} & 9.9031 \\ \frac{1}{197.6140} & 197.6140 \end{bmatrix}\begin{Bmatrix} a_0 \\ a_1 \end{Bmatrix} = \begin{Bmatrix} .0268 \\ .0268 \end{Bmatrix}$$

From which

$a_0 = 0.5055$ and $a_1 = 0.00026$

4 AlManshia Bridge

The case studied is the deck on the first span of the new Blue Nile Bridge (ALManshia Bridge) which is simply supported pre-stressed concrete bridge of total span (36 m c/c), consisting of two carriage ways each having (8.25m) traffic lanes, and two footways (1.75m) width each.

The total width of the bridge deck is (20.5m), the system consist of (10) parallel longitudinal T-beams with (0.2m) thick slab cast in situ and (0.5 m) central reserve. The system is shown in Figure 4.

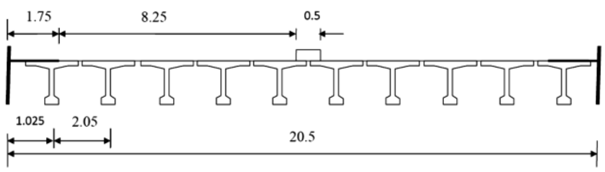

Figure 4: ALManshia Bridge Cross Section

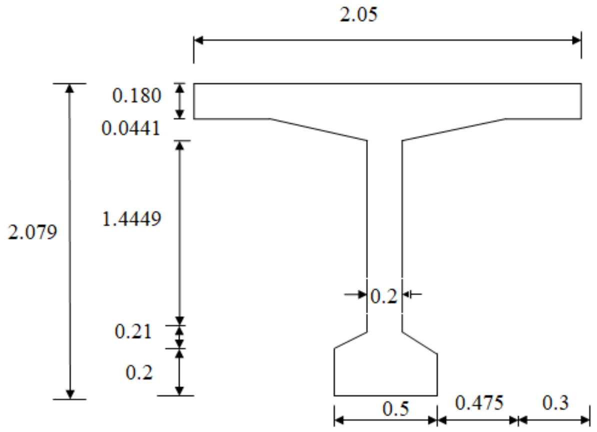

Figure 5: ALManshia T-beam Section

(all dimensions are in meters)

4.1 Case Study model

The bridge deck is modeled as space frame as shown in Figure 6.

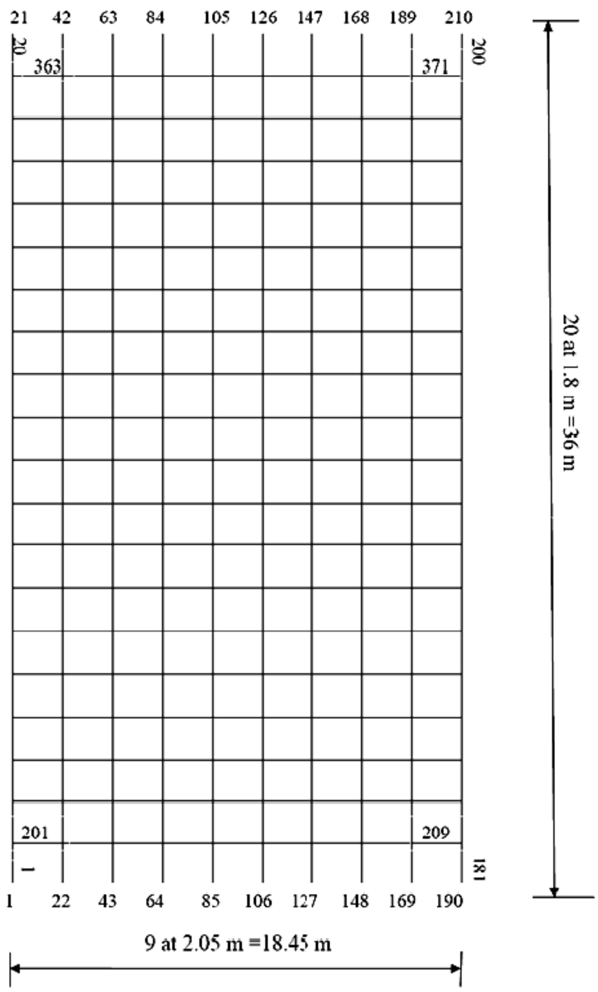

Figure 6: ALManshia Space Frame Model.

4.2 Section Properties

Section properties of the bridge members are as follows:

Longitudinal members:

Ix=.01413 m4

Iy=.28481 m4

Iz=.65889 m4

A= 1.2911 m2

Transverse members:

Ix=.00444. m^4

Iy=.0972 m^4

Iz=.0012 m^4

A= .36 m^2

4.3 Damping matrix

According to EC8 - EN 1998-2 a damping of ξ= 0.05 will be considered [19].

$$C = a_0 M + a_1 K \qquad (8)$$

a_0, a_1 proportional factors determined from specified damping ratios ξ_i and ξ_j for the ith and jth modes, respectively.

$$\frac{1}{2}\begin{bmatrix} \frac{1}{\omega_i} & \omega_i \\ \frac{1}{\omega_j} & \omega_j \end{bmatrix} \begin{Bmatrix} a_0 \\ a_1 \end{Bmatrix} = \begin{Bmatrix} \xi_i \\ \xi_j \end{Bmatrix} \qquad (9)$$

For this case we take 1st mode and 5th mode

From modal analysis ω_1 =17.714 rad/sec

and $\omega_5 = 38.30\ rad/sec$

Substitute in Eq (9)

$$\frac{1}{2}\begin{bmatrix} \frac{1}{17.714} & 17.714 \\ \frac{1}{38.30} & 38.30 \end{bmatrix} \begin{Bmatrix} a_0 \\ a_1 \end{Bmatrix} = \begin{Bmatrix} .05 \\ .05 \end{Bmatrix}$$

From which

$a_0 = 1.2112$ and $a_1 = 0.0018$

4.4 Design Spectrum for Soil at Central Khartoum

To consider the local situation of Khartoum the following paper will be used:

DEVELOPMENT OF DESIGN RESPONSE SPECTRAL FOR CENTRAL KHARTOUM, SUDAN [7]

Y. E-A. Mohamedzein · J. A. Abdalla · A. Abdelwahab

The following conclusions which are drawn from the paper are considered in the calculations:

Rifts and faults in North Kordofan State have been very active recently. They are capable of producing damaging earthquakes in the presumably low risk area of Khartoum. A bedrock Peak Ground Acceleration PGA of 0.1 g is anticipated in Central Khartoum with a 10% probability of exceedance in 50 years.

The results indicated amplification of ground motion of up to 4.93.

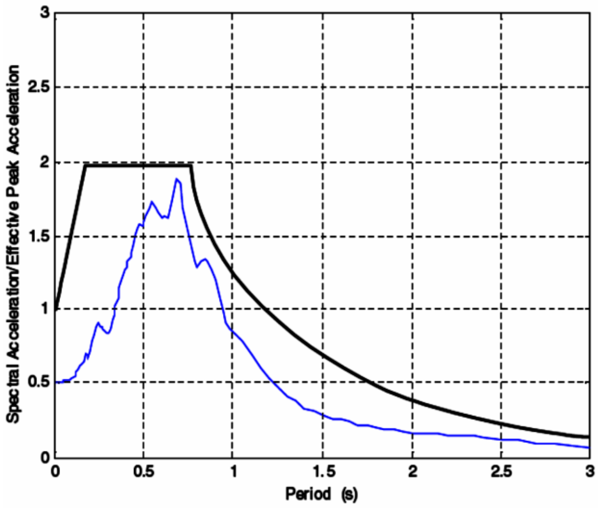

Figure 7: Average Acceleration Spectra Ratio and Design Spectrum for Soil at Central Khartoum

Peak Ground Acceleration PGA= 0.1g*4.93=0.493g

The response of the bridge will be determined separately for each direction (lateral, vertical and longitudinal).

According to EC 8 all responses can be combined as follows [19]:

a) EEdx + 0.3 EEdy+0.3 EEdz

b) EEdy + 0.3 EEdx+0.3 EEdz

c) EEdz + 0.3 EEdx+0.3 EEdy

An envelope for all cases is determined and drawn in graphs

5 Artificially Generated Earthquakes

Depending on response spectrum of soil at central Khartoum shown in section 4.4 and using SIMQKE program the following earthquakes shown in Figure 8 were artificially generated with different spectrum ratio in which earthquake 1 with one third response spectrum, earthquake 2 with two third response spectrum and earthquake 3 with full response spectrum.

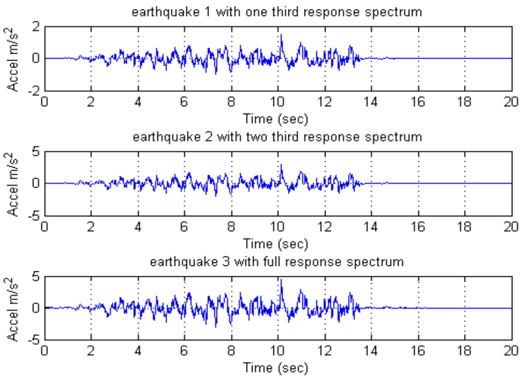

Figure 8: Artificially Generated Earthquakes

6 Results for Burri Bridge

6.1 Modal Analysis

Modal Analysis shows that the Natural Vibration Frequency for the bridge is 1.576 cycle/sec and the circular frequency is 9.90 rad/sec which is slighty greater than the one 1.36 cycle/sec measured by the dynamic test conducted on the year 2005 by the Chinese Longian company.

6.2 Time History and Response Spectrum analysis results

The following abbreviations will be used in the contexts:

T.H: Time History, R.S: Response Spectrum

S.F: Shear Force, B.M: Bending Moment

The values of Maximum Dynamic Y-Displacement, S.F and B.M of the cantilever beam due to earthquake 1 and one third response spectrum are shown in Figures 9-11

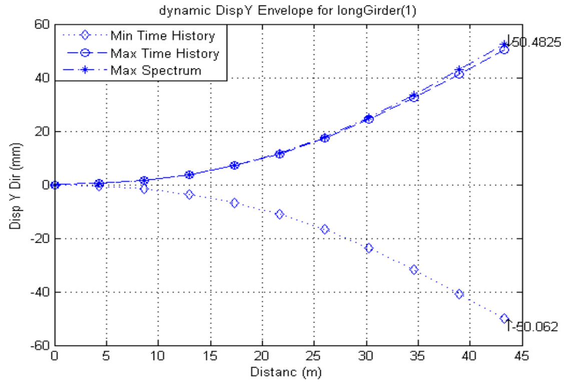

Figure 9: Maximum Dynamic Y-Displacement (0.333 spectrum ratio).

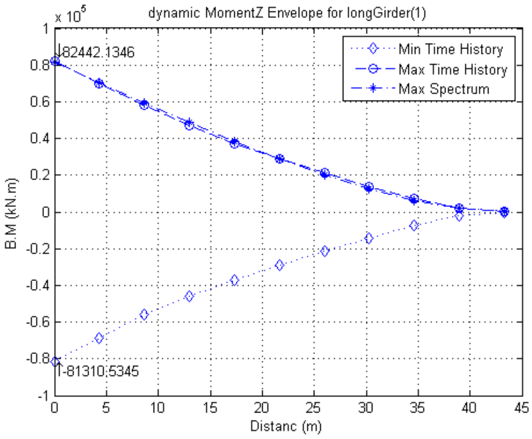

Figure 10: Maximum Dynamic Bending Moment

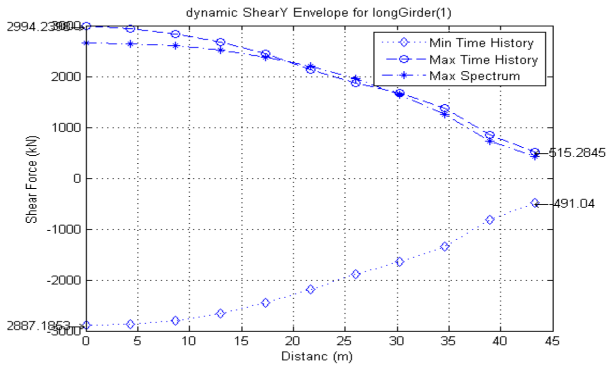

Figure 11: Maximum Dynamic Shear Force (0.333 spectrum ratio).

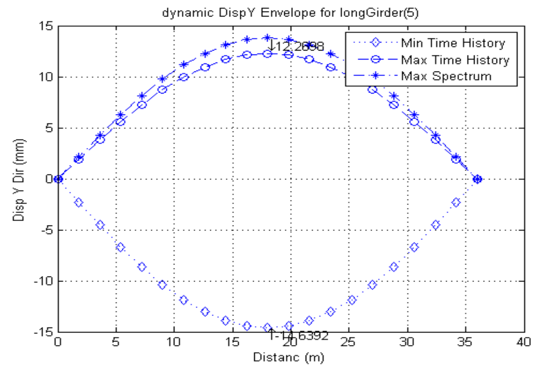

Figure 12: Y-Displacement for Longitudinal Girder(5) of Al Manshia Bridge.

The variation of response with different values of spectrum ratio is demonstrated in Table 1

Table 1 Variation of Response with Different Values of Spectrum Ratios

Spectrum ratio	End Y-Displ (mm)	Root Shear (kN)	Root Moment (kN.m)
0.333	52.42	2652.55	81727.62
0.667	104.82	5304.40	163430.48
1	157.24	7956.95	245158.16

7 Results for Al Manshia Bridge

7.1 Modal Analysis

Modal Analysis shows that the Natural Vibration Frequency for the bridge is 2.82 cycle/sec and the circular frequency is 17.71 rad/sec.

7.2 Time History and Response Spectrum analysis results

The values of Maximum Dynamic Y-Displacement, Y-S.F and Z-B.M for Longitudinal Girder (5) of Al Manshia Bridge due to earthquake 1 and one third response spectrum are shown in Figures 12-14

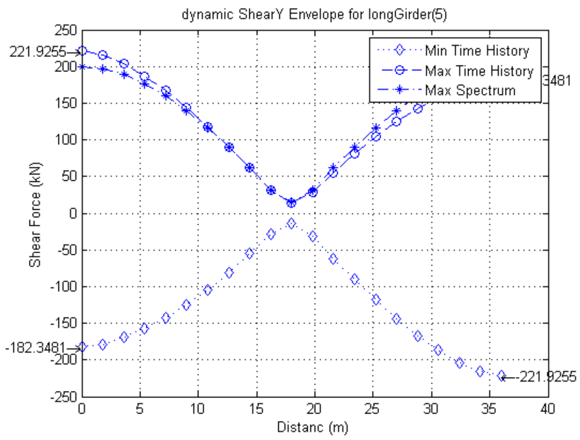

Figure 13: Y-S.F for Longitudinal Girder(5) of Al Manshia Bridge.

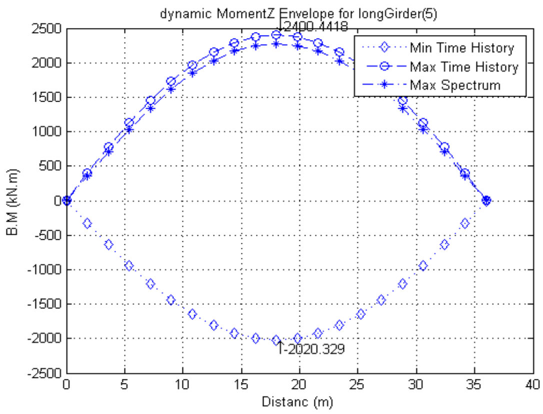

Figure 14: Z-B.M for Longitudinal Girder(5) of Al Manshia Bridge.

8 Discussion of Results

1 Modal Analysis results in section 6.1 shows that the rigidity of actual structure is lower than the theory rigidity. It shows that the rigidity of the bridge has reduced under the long-playing using load.

2 Table 1 show that the response of the bridge in terms of Y-Displacement, Root Shear and Root Moment are directly proportional to the response spectrum ratio used.

3 Since the response spectrum depends on Peak Ground Acceleration (PGA) it means that the response of the bridge is directly proportional to the PGA.

4 According to this result the other Case Study was analyzed with 0.333 of response spectrum i.e. with PGA= 0.333× (0.1g×4.93) = 0.164 g.

5 The following calculations are to obtain the PGA which limits bending moment to ultimate moment capacity for Burri Bridge

Ultimate B. M. Capacity = 396720 kN.m
Dead Load B. M. = 323187 kN.m
Remaining B. M. Capacity = 73533 kN.m

From Seismic Analysis:

0.164 g ⟶ 82442 kN.m
PGA ⟶ 73533 kN.m

PGA = 0.146 g

6 The following calculations are to obtain the PGA which limits shear the force to ultimate shear capacity for Burri Bridge.

Ultimate S. F Capacity = 19044 kN
Dead Load S.F = 15936 kN
Remaining S.F Capacity = 3108 kN

From Seismic Analysis:

0.164 g ⟶ 2994 kN
PGA ⟶ 3108 kN

PGA = 0.17 g

7 The following calculations are to obtain the PGA which limits the vertical displacement to a maximum of L/800 for Burri Bridge;

$L/800 = 43.3\times10^{3}/800 = 54.125$ mm

0.164 g ⟶ 52.40 mm
PGA ⟶ 54.125 mm

PGA = 0.169 g

8 Same calculations was done for the case of Almanshia Bridge.

Table 2shows the results for the two bridges

Table 1: Peak Ground Acceleration which limits response to ultimate capacity

	Burri Bridge	ALManshia Bridge
Moment	0.146 g	0.60 g
Shear	0.17 g	0.35 g
Displacement	0.169 g	0.55 g

9 General conclusions for the two case studies

1. Bridge responses in terms of Displacements, Shear and Moments are directly proportioned to the peak ground acceleration (PGA).
2. Time History Analysis and Response Spectrum Analysis almost gave the same results. The results of Time History Analysis compared with the results of Response Spectrum Analysis were as follows:

(I) **Case Study One (Burri Bridge):**

There was a very close agreement for bending moment results between the two methods. The maximum Time History Displacement in Y-Direction, maximum shear force and maximum bending moment were 50.48 mm, 2994.2 kN and 82442.13 kN.m respectively. The percentage differences as compared with response spectrum were 3.84 %, 11.41 %, and 0.87 % respectively.

(II) **Case Study Two (AlManshia Bridge):**

There was also a close agreement for bending moment results between the two methods. The maximum Time History Displacement in Y-Direction, maximum shear force and maximum bending moment were 12.27 mm, 221.93 kN and 2400.44 kN.m respectively. The percentage differences as compared with response spectrum are 10.96 %, 9.92 %, and 5.08 % respectively.

10 Conclusions for Case Study One Burri Bridge:

1. The PGA which limits moment to moment capacity was equal to 0.146 g.
2. The PGA which limits shear to shear capacity was equal to 0.17g.
3. The PGA which limits vertical displacement to maximum displacement was equal to 0.169g.

11 Conclusions for Case Study Two AlManshia Bridge:

1. The PGA which limits moment to moment capacity is equal to 0.60 g.
2. The PGA which limits shear to shear capacity was equal to 0.35g.
3. The PGA which limits vertical displacement to maximum displacement was equal to 0.55g.

12 Recommendations &Future research:

The following recommendations are directly affiliated with this thesis;

1. Due to the ease of implementation for the response spectrum method it is recommended for the evaluation of seismic effects on the bridges.
2. Where nonlinearity may exist such in piers and abutment it is recommended to use nonlinear time history analysis to evaluate seismic effects.
3. It is recommended that design response spectrum is to be used for other areas of Sudan.
4. The following topics could be suggested for future studies
   a) Investigating of other components of the bridge such as piers and abutments for seismic effects.
   b) Investigating of the methods of pushover analysis and comparing them with nonlinear time history analysis.